Proceedings of the Ninth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Society
14-16 April, 2011, Auckland, New Zealand

Design of UFP-coupled post-tensioned timber shear walls

M.P. Newcombe[1], D. Marriott[2], W. Y. Kam[1], S. Pampanin[1] & A.H. Buchanan[1]

[1]*Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand.*

[2]*Holmes Consulting Group Ltd, Wellington, New Zealand*

ABSTRACT: Recent advances in timber design at the University of Canterbury have led to new structural systems that are appropriate for a wide range of building types, including multi-storey commercial office structures. These buildings are competitive with more traditional construction materials in terms of cost, sustainability and structural performance. This paper provides seismic design recommendations and analytical modelling approaches, appropriate for the seismic design of post-tensioned coupled timber wall systems. The models are based on existing seismic design theory for precast post-tensioned concrete, modified to more accurately account for elastic deformation of the timber wall systems and the influence of the floor system. Experimental test data from a two storey post-tensioned timber building, designed, constructed and tested at the University of Canterbury is used to validate the analytical models.

1 INTRODUCTION

High performance solid timber wall systems have been developed at the University of Canterbury, in collaboration with the Structural Timber Innovation Company (STIC Ltd). Post-tensioning tendons or bars are used to connect large wall sections (see Fig. 1a) to a concrete foundation or basement level. The wall sections are constructed from engineered wood product, such as Glulam, Laminated Veneer Lumber (LVL) or Cross Laminated Timber (CLT). Post-tensioned walls provide an efficient structural system, suitable for a wide range of building types, including commercial structures, and have the potential to compete with existing forms of construction in terms of cost, versatility, structural performance and energy efficiency (Smith *et al.*, 2009).

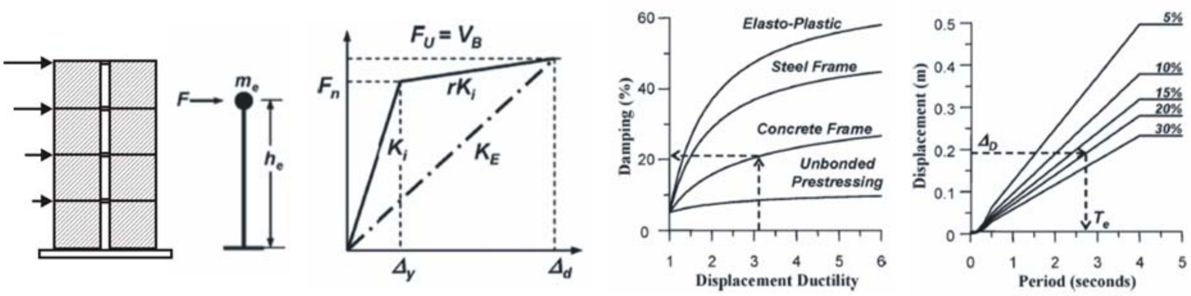
The concept of post-tensioned timber (Palermo *et al.*, 2005) was adapted from the PRESSS-technology, developed for jointed ductile pre-cast concrete systems (Kurama *et al.*, 1999; Priestley *et al.*, 1999; Rahman and Restrepo, 2000). For seismic design, the combination of timber and post-tensioning is particularly efficient since it avoids loss of stiffness and strength, and potential brittle failure modes that can occur in traditional timber connections. Previous research (Iqbal *et al.*, 2007; Smith *et al.*, 2007; Newcombe *et al.*, 2010) has shown that post-tensioned timber wall elements respond essentially elastically to even severe earthquake loading, and due to the restoring action provided by the post-tensioning, residual deformations are insignificant. With the addition of coupling elements between walls, such as U-shaped flexural plates (Kelly *et al.*, 1972), the overall overturning moment as well as energy dissipation capability of the system can be significantly enhanced. This was shown by Priestley *et al* (1996) for precast concrete and confirmed for the timber emulative solutions (Iqbal *et al.*, 2007; Smith *et al.*, 2007; Newcombe *et al.*, 2010). Hence, the system fits well into recent Performance-Based Seismic Engineering (PBSE) design approaches (Christopoulos and Pampanin, 2004) to minimise structural damage, residual deformation, downtime and associated financial losses.

There has been extensive work on the lateral force and detailed design of couple reinforced and precast concrete wall systems (Kurama, 2002; Sullivan *et al.*, 2006; Priestley *et al.*, 2007; Jiang and Kurama, 2010). Furthermore, for precast concrete comprehensive design information has been provided in the NZCS PRESSS Handbook (Pampanin and Marriott, 2010). However, these design approaches require modification so they can be extended to timber walls, which possess unique material properties.

2 LATERAL FORCE DESIGN

A simplified single degree of freedom (SDOF) lateral force design methodology is used to determine the actions on the coupled wall systems due to earthquake. The seismic design can be either an equivalent static force-based design (NZS1170.5:2004) or a displacement-based design (Priestley *et al.*, 2007). For both procedures, the fundamental period should be determined considering an allowable design displacement for a given structural performance level, with a corresponding earthquake intensity. If an equivalent static force-based design procedure is applied, current force reduction factors (NZS1170.5, 2004) are not appropriate. Expressions must be used which consider the unique energy dissipative characteristics of a rocking-dissipative wall system (with a flag-shape hysteresis loop), such as those proposed by Priestley *et al* (2007). For this paper, a displacement-based methodology is considered, as it is the most direct method to determine lateral forces while addressing structural performance levels.

Priestley *et al* (2007) provides details on performing a displacement-based lateral force design of coupled walls, as illustrated in Figure 1. Key aspects of the procedure that require further considerations for coupled post-tensioned timber walls are the displacement profile, yield displacement and the equivalent viscous damping.



a) b) c) d)

Figure 1. Displacement-based design (Priestley *et al.*, 2007): a) SDOF representation of wall system b) Effective stiffness c) Equivalent viscous damping d) Displacement spectrum

2.1 The displacement profile

The displacement profile (or mode shape) defines the effective mass, height and design displacement of the equivalent SDOF (see Fig. 1a). A linear displacement profile is assumed for the displacement-based design of post-tensioned precast concrete walls (Priestley *et al.*, 2007; Pampanin and Marriott, 2010). However, for timber this may not be strictly appropriate because the elastic deformation of the timber walls is more significant.

The wall displacement is the sum of the connection deformation, flexural deformation, and shear deformation, as shown in Fig. 2. If the elastic deformation of the wall elements is significant and a linear displacement profile is assumed, the displacement-based design may be unconservative. If shear deformations are dominant, the drift demand on the first floor will be critical, thus increasing the overall seismic demand. If flexural displacements are dominant, the critical drift demand will be on the top floor; again increasing seismic demand.

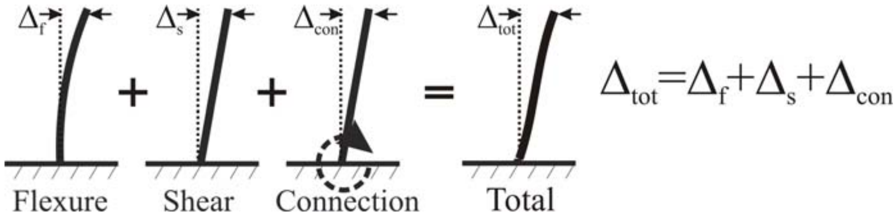


Figure 2. Displacement components of wall elements

The shape of the displacement profile depends on the proportion of the total wall displacement from the connection (**R_{con}**), the aspect ratio of the wall (**A_r**)and the proportion (**β_{CB}**) of the overturning moment (**M_{OTM}**) provided by the coupling elements defined in Equations 1, 2 and 3 respectively.

$$R_{con} = \frac{\Delta_{con}}{\Delta_{tot}}; \qquad A_r = \frac{H_1}{l_w}; \qquad \text{and} \qquad \beta_{CB} = \frac{M_{CB,B}}{M_{OTM}} \qquad (1), (2) \text{ and } (3)$$

where $\boldsymbol{l_w}$ = the length of each wall, $\boldsymbol{H_1}$ = the interstorey height and $\boldsymbol{M_{CB,B}}$ = the base moment provided by the coupling elements.

If **R_{con}** is large, the connection deformation is dominant and hence the displacement profile will tend to be linear. As **A_r** increases, the relative significance of the flexural deformation increases, which affects the shape of the displacement profile. Furthermore, if **β_{CB}** is large, the wall-system will respond with a predominantly shear-type displacement profile.

A sensitivity study was performed using realistic upper and lower bounds for **R_{con}**, **A_r**, and **β_{CB}** to determine whether a linear displacement profile was appropriate for design. **R_{con}** varied between 0 and 0.8. Often wall designs are governed by serviceability displacement limitations. For such designs, the wall-base moments may not achieve decompression, resulting in effectively no connection rotation. **A_r** varied between 1 and 2.5 to match with realistic wall geometries. **β_{CB}** varies from 0.1 to 0.5. If **β_{CB}** is larger than 0.5, re-centering of the wall system will not occur, which is not considered as design possibility.

The results of the sensitivity study are presented for a four storey coupled wall system in Fig. 3. The most significant deviation from a linear displacement profile occurs when **R_{con}**, **A_r**, and **β_{CB}** are 0, 2.5 and 0.1 respectively. For this case, if a linear displacement profile is assumed, the base shear according to displacement-based design will be under estimated by 17%. Conversely, if the wall system is designed for the base shear for a linear displacement profile, the peak drift will be 17% higher than expected. For a simplified design procedure, this error is acceptable, given that lateral strength provided by the floor and gravity systems, which may be significant (Newcombe *et al.*, 2010) but is currently ignored.

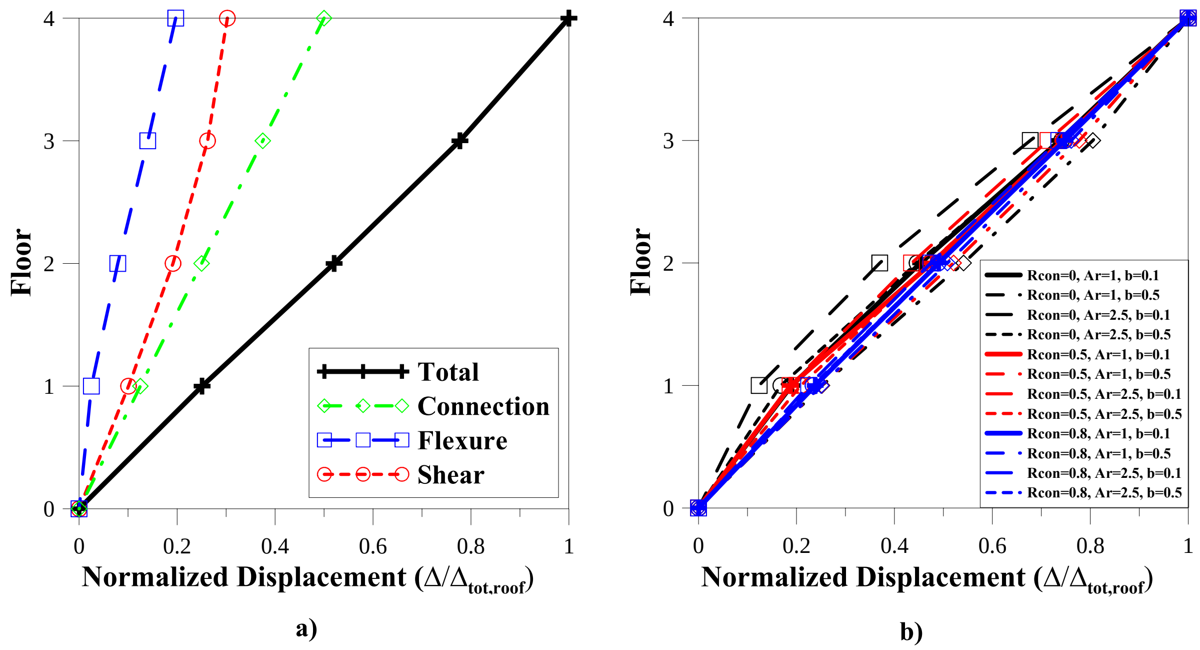


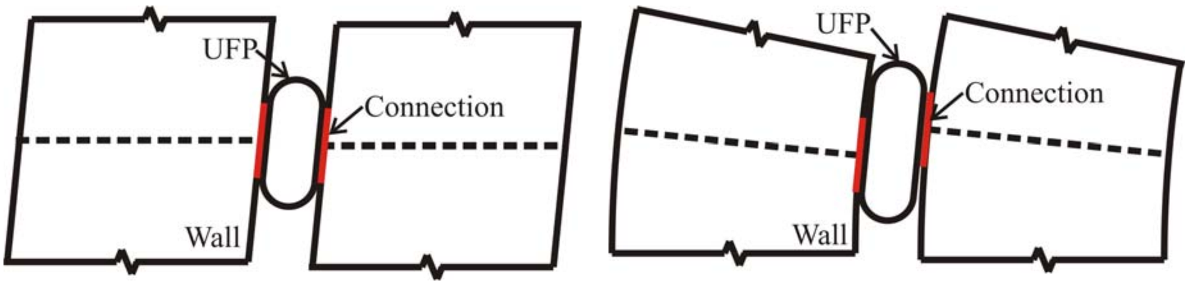
Figure 3. Displacement profile for four story coupled timber walls: a) Displacement contributions (R_{con}=0.5; A_r,=1; β_{CB}= 0.5) b) Sensitivity study

2.2 The yield displacement

For the design of reinforced concrete walls the yield displacement is essentially independent of the strength of the wall system (Priestley *et al.*, 2007). For timber, this is not the case because the elastic deformation of the wall elements is significant, so the displacement-based design procedure may need to be iterative. The yield displacement must be guessed initially and then checked at the end of the design process. Provided a conservative (higher) initial estimate of the yield displacement of the wall system is made, the design process need not be repeated. Over estimating the yield displacement, results in reduced ductility, less equivalent viscous damping and conservative seismic demands.

Potential contributions to the yield displacement of the coupled wall system are the connection deformation (between the UFP devices and the wall) and the elastic deformation of the walls elements. According to Kelly *et al* (1972), the UFP devices begin to yield instantly when displaced, or in other words, have effectively infinite initial stiffness. However, the connections used to attach the UFPs to the walls may deform resulting in delayed activation of the UFPs. The contribution of the UFP connection deformation to yield displacement of the system can be approximated in Equation 5. Assuming that the UFP devices are connected to the walls using screws or nails, it is reasonable to assume an upper bound connection deformation, Δ_{ufp}, of approximately 1mm, as inferred from NZS3603 (1999).

In Figure 4, the effect of the elastic deformation components on the UFP couplers is illustrated. It is evident that shear deformation can occur without activating the UFPs. However, activation will only be avoided if there are slight vertical shear distortions in the wall elements or the UFP connections are able to rotate slightly. Conservatively, it is considered that this is case. From finite element analysis (see section 4), it was shown that axial deformations of the walls also delayed the activation of the UFP couplers. As an upper-bound, the axial deformations contributed an additional 50% to the system yield displacement. Flexural deformation of the wall elements will activate the UFP devices, and hence, should not contribute the yield displacement.



a) b)

Figure 4. The effect of elastic deformation on the UFP couplers: a) Shear distortion b) Flexural distortion

Therefore, to determine the yield displacement of the wall system, $\mathbf{\Delta_{y,e}}$, the UFP connection deformation, $\mathbf{\Delta_{ufp,e}}$, and the elastic deformation of the wall, $\mathbf{\Delta_{w,e}}$, should be considered:

$$\Delta_{y,e} = \Delta_{ufp,e} + \Delta_{w,e} \tag{4}$$

$$\Delta_{ufp,e} = 2H_e \frac{\Delta_{ufp}}{l_w} \tag{5}$$

where $\boldsymbol{H_e}$ = the effective height and $\mathbf{\Delta_{ufp}}$ = the connection deformation between the UFP device and the wall (possibly 1mm).

As mentioned above, to determine the wall deformation at the yield point, the strength of the building is required. Hence, the yield point of the wall system can not be accurately defined until the detailed design is completed. Using the sensitivity study in the previous section, the shear distortion can contribute to between 10 to 60% of the elastic deformation of the wall elements. For example, if the connection rotation contributes to half of the total wall deformation, then the yield displacement can be between 5 to 30% of the total wall deformation, which will give a ductility of between 20 and 3 respectively. Hence, significant design errors are possible if the yield point of the wall system is not correctly evaluated.

Conservative estimates of the shear deformation can be made during the displacement-based design procedure, avoiding iteration of the detailed wall design. The following expression can be applied at the end of the displacement-based design iteration to conservatively estimate the wall deformation:

$$\Delta_{w,e} \approx 0.4 \frac{\left(1-\beta_{CB}\right)}{\gamma_{LS} A_r^2} \frac{V_b H_e^3}{EI} \tag{6}$$

where $\boldsymbol{\gamma_{LS}}$ = 1.0 and 1.25 for serviceability and ultimate limit state design respectively, $\boldsymbol{V_b}$ = the base shear and $\boldsymbol{EI}$ = the flexural stiffness of one wall element.

2.3 Computation of the equivalent viscous damping

To compute the equivalent viscous damping, $\boldsymbol{\xi_{eq}}$, for the displacement-based design procedure, both the elastic (or intrinsic) damping, $\boldsymbol{\kappa\xi_{el}}$, and the hysteretic damping, $\boldsymbol{\xi_{hyst}}$, must be defined, where $\boldsymbol{\kappa}$ is a correction factor to account for secant stiffness for the equivalent SDOF used in displacement-based design (Priestley *et al.*, 2007).

Previous researchers on wood structures have recommended elastic damping values of between 2% and 5% of critical damping. Filiatrault *et al* (2002) and Christovasilis *et al* (2007) obtained a minimum values of 3% and 5% respectively from experimentation on light timber frame houses. More recently, Pang and Rosowsky (2007) suggest that 5% elastic damping is appropriate for medium rise light timber frame. However, light timber frame and coupled heavy timber walls (consisting of laminated timber) are very different structural systems, and consequently may provide significantly different levels of elastic damping. Shake-table tests performed on post-tensioned timber walls by Marriott (2009) indicate lower damping values of approximately 2%. Hence, for conservative design it is proposed that the elastic damping for coupled timber walls should be 2%. Because the secant and tangent stiffness of the system is comparable (due to a high post-yield stiffness) it is suggested that the elastic damping will not significantly reduce with ductility, as suggested by Priestley *et al* (2007). Hence, the correction factor, **κ,** can be taken as 1.0.

The hysteretic damping of the wall system, $\xi_{eq,sys}$, can be determined by using the area-based damping (the area within a complete loop) of the hysteresis. For precast concrete walls, the NZCS PRESSS Design Handbook (2010) uses empirical expressions for damping calibrated from extensive time history analysis on SDOF systems. However, these expressions are too conservative because they are derived for non-re-centering systems, which tend to be less centrally orientated and provide less damping. Priestley *et al* (2007) suggests that for a flag-shaped hysteresis there is essentially no difference between the hysteretic damping derived from an area-based approach or that calibrated from time-history analysis. Therefore, Equation 8 is suggested, based on the flag-shaped hysteresis. The elastic and hysteretic components are scaled according to their respective contributions to the overturning moment capacity (Priestley *et al.*, 2007). Hence:

$$\xi_{eq,sys} = \xi_{el} + \xi_{hyst} \text{ And } \xi_{hyst} = \frac{2\beta_{CB}(\mu - 1)}{\pi\mu(1 + r(\mu - 1))} \qquad \text{(7) and (8)}$$

where **μ** is the system ductility, defined as the design displacement, Δ_d, divided by the yield displacement, $\Delta_{y,e}$, and **r** is the post-yield stiffness of the system. Conservatively, **r** can be taken as 0.06. These equations result in a maximum achievable damping for a re-centering wall system of approximately 20%.

3 DETAILED DESIGN

A procedure for detailed design of UFP coupled precast concrete walls is provided in the NZCS PRESS Design Handbook (2010). Again, slight modifications are required due to the specific properties of timber.

3.1 Computation of the imposed rotation

To evaluate the moment at the base of the walls, the connection rotation, θ_{imp}, must be determined. θ_{imp} is calculated by taking the allowable structural drift for a given design limit state, θ_D, and subtracting the elastic deformation due to flexure and shear, θ_f and θ_s respectively. The elastic deformation can be approximated as:

$$\theta_f + \theta_s \approx V_b \frac{(1 - \beta_{CB})H_e^{\,2}}{6EI} + \frac{\overline{V_s}}{2GA_s} \qquad (9)$$

where $\overline{V_s}$ is the average storey shear between the base and the effective height and can conservatively be taken as 85% of the base shear.

Therefore:

$$\theta_{imp} = \theta_D - (\theta_f + \theta_s) \qquad (10)$$

3.2 Evaluation of the wall-base connection moment

Procedures exist to determine the connection moments (Newcombe *et al.*, 2008), which are based on similar procedures for precast concrete (NZS3101, 2006). Firstly, the Monolithic Beam Analogy (Pampanin *et al.*, 2001; Palermo, 2004) is used to determine the strain in the timber, $\boldsymbol{\varepsilon_t}$, as follows:

$$\varepsilon_t = \left(\frac{3\theta_{imp}}{H_e} + \phi_{dec} \right) c \qquad (11)$$

where $\boldsymbol{\phi_{dec}}$ is decompression curvature of the wall and **c** is the depth of the compression region (or neutral axis depth). The stress in the timber at the extreme fibre, $\mathbf{f_t}$, is calculated using a calibrated connection modulus, $\mathbf{E_{con}}$:

$$f_t = E_{con}\varepsilon_t \text{ And } E_{con} = 0.6E_t \qquad (12) \text{ and } (13)$$

where $\mathbf{E_t}$ is mean elastic modulus of the timber excluding end effects.

A linear stress block is appropriate for the compression region, because for most designs, the timber remains elastic. For further information refer to Newcombe *et al* (2008).

3.3 Evaluation of the total over-turning moment

The total overturning moment (**OTM**) of the wall system is defined in Equation 14. The coupling action of the UFP devices causes variations of axial force in each wall, which results in variations of the moment provided by the base-connections of each wall ($M_{wall,1}$ and $M_{wall,2}$). Furthermore, the variation of axial force in each wall results in variations of the length of the compression region (or neutral axis depth) in each wall.

For precast concrete design (Pampanin and Marriott, 2010) the variation in the neutral axis depth between each wall is small, and hence, it is reasonable to assume that lever-arm used for computation of the moment contribution from UFP is equal to the centreline distance between the walls. However, timber connections have approximately 20% of compressive stiffness of an equivalent concrete wall. This results in significant variations in the neutral axis depth of each wall, which reduces the moment contribution from the UFP devices.

$$OTM = M_{wall,1} + M_{wall,2} + V_{ufp}\left(l_{cl} - \frac{c_1 - c_2}{3} \right) \qquad (14)$$

where $\mathbf{l_{cl}}$ is distance between the centreline walls, $\mathbf{c_1}$ and $\mathbf{c_2}$ are the neutral axis depths of the wall subject to minimum and maximum compression respectively.

4 TIME-HISTORY ANALYSIS VALIDATION

The above design approach was validated using time-history analysis (THA), with 15 spectrum-compatible (NZS1170.5, 2004) natural earthquake motions, for a case study three-storey coupled wall system. The finite element model of the wall system is described in Fig. 5a. The wall elements include flexural and shear deformation. Axial springs are used to represent the UFP couplers. The UFP-springs yield at 1mm vertical displacement. The base connections are modelled using multi-spring elements.

The yield displacement of the wall system was predicted using Equations 4, 5 and 6. The yield displacement, obtained from a push-over analysis of the finite element model, is slightly less than predicted (see Fig. 5b), due to conservatism built into the equations.

For the THA, Fig. 5c shows that drift demand is slightly over-estimated by the design. This is due to the slight under estimation of the system ductility, which resulted conservative system damping, given by Equation 8. The shear forces induced in the wall elements is underestimated by the procedure (see Fig. 5d) due to higher mode effects. Dynamic amplification factors appropriate for coupled timber walls are required, and will be the topic of future research. However, the size of the wall elements is usually governed by deflection limitations. Hence, highly conservative dynamic amplification factors can be applied to check the strength of the walls.

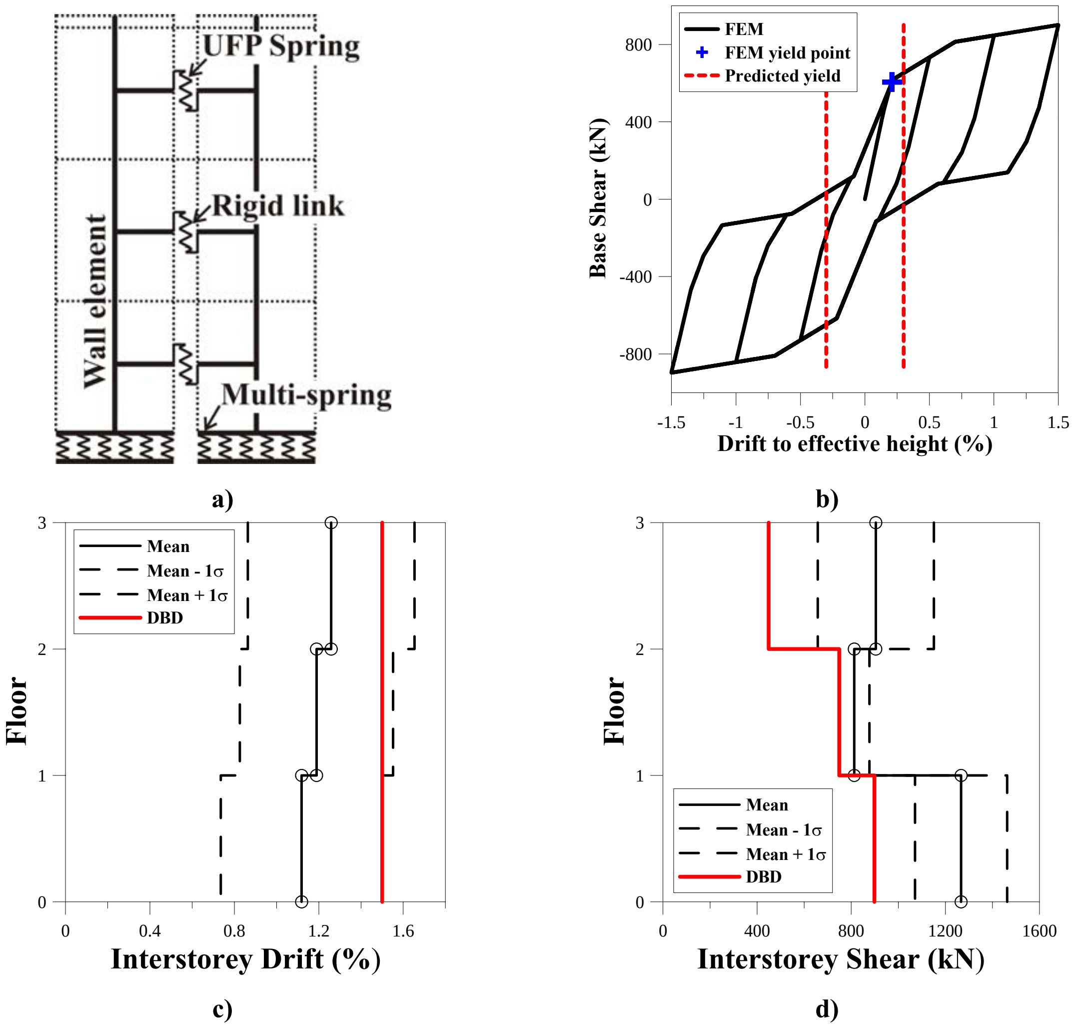


Figure 5. Design verification using time-history analysis: a) Finite element model b) Drift demand c) Storey Shear forces

5 CONCLUSIONS AND FURTHER RESEARCH

This paper provides seismic design recommendations and analytical modelling approaches for multi-storey post-tensioned coupled timber wall systems.

For lateral force design, it is determined that a linear displacement profile is appropriate, that shear deformation and UFP connection slip contribute to the yield displacement, and that the equivalent viscous damping can be determined using existing theory. Because the yield displacement depends on elastic deformation of the wall elements, iteration of the lateral force design may be required. To minimise iterations conservative estimates of the yield displacement have been proposed.

For the detailed design of the walls, existing approaches for similar walls in precast concrete can be applied. However, special consideration of the elastic deformation and stress/strain within the connection are required. For a given design drift limit, the elastic deformation of the wall reduces the allowable connection rotation (which defines the strength of wall system). Therefore, equations are proposed to conservatively estimate the elastic deformation. Within the compression region at the base of the wall, a linear stress-strain law is applied, where the strain is estimated using existing theory, termed the Monolithic Beam Analogy.

Further research on coupled timber wall systems is required to refine the design procedure. Time-history analysis should be performed on a wide range of wall system geometries, with different degrees of coupling ($\boldsymbol{\beta}_{CB}$ values) and for different governing performance limit states. The dynamic amplification of storey shears and moments needs evaluation and implementation into the design procedure. In addition, the contribution of the gravity system to the lateral strength of the wall system may be significant and requires further evaluation. Some of these issues will be addressed in future publications.

REFERENCES:

Christopoulos, C., and Pampanin, S. 2004. Towards performance-based design of MDOF structures with explicit consideration on residual deformations, Special Issue on "Performance-Based Seismic Design". *ISET Journal of Structural Engineering,* Vol. **41: 1**, pp 53-73.

Christovasilis, I. P., Filiatrault, A., and Wanitkorkul, A. 2007. Seismic Testing of a Full-Scale Two-Story Light-Frame Wood Building : NEESWood Benchmark Test. *Report NW-01*, NEES Wood, Buffalo, USA.

Filiatrault, A., ASCE, M., Fischer, D., Folz, B., and Uang, C. M. 2002. Seismic Testing of Two-Story Woodframe House: Influence of Wall Finish Materials. *Journal of Structural Engineering*, Vol. **10: 128**.

Iqbal, A., Pampanin, S., Buchanan, A., Fragiacomo, M., and Palermo, A. 2007. Application of Hysteretic Dampers in LVL Coupled Walls for Improved Seismic Performance. *Proceedings, 8th Pacific Conference of Earthquake Engineering, Singapore*.

Jiang, H., and Kurama, Y. C. 2010. Analytical modeling of medium-rise reinforced concrete shear walls *PCI Journal*, Vol. **107: 4**, pp. 400-410.

Kelly, J. M., Skinner, R. I., and Heine, A. J. 1972. Mechanism of Energy Absorption in Special Devices for Use in Earthquake Resistant Structures. *Bulletin New Zealand Society of Earthquake Engineering*, Vol. **Vol. 5: 3**, pp. 63-88.

Kurama, Y. C. 2002. Hybrid post-tensioned precast concrete walls for use in seismic regions. *PCI Journal*, Vol. **47: 5**, pp. 36-59.

Kurama, Y. C., Sause, R., Pessiki, S., and Lu, L. 1999. Lateral load behavior and seismic design of unbonded post-tensioned precast concrete wall. *PCI Journal*, Vol. **46: 4**, pp. 622-633.

Marriott, D. 2009. "The development of high-performance post-tensioned rocking systems for seismic design of structures," Doctorate thesis, University of Canterbury, Christchurch, New Zealand.

Newcombe, M. P., Pampanin, S., Buchanan, A., and Palermo, A. 2008. Section Analysis and Cyclic Behavior of Post-Tensioned Jointed Ductile Connections for Multi-Storey Timber Buildings. *Journal of Earthquake Engineering*, Vol. **12: Special Issue**, pp. 83-110.

Newcombe, M. P., Pampanin, S., and Buchanan, A. H. 2010. Global Response of a Two Storey Pres-Lam Timber Building. *New Zealand Society of Earthquake Engineering Conference*, Wellington, New Zealand, pp. 8.

NZS1170.5. 2004. *Structural Design Actions - Part 5 - Earthquake Actions*, New Zealand Standards, Wellington.

NZS1170.5:2004. *Structural Design Actions - Part 5 - Earthquake Actions*, New Zealand Standards, Wellington.

NZS3101. 2006. *Concrete Structures Standard: Appendix B: Special Provisions for Seismic Design of Ductile Jointed Precast Concrete Structural Systems*, New Zealand Standards, Wellington.

NZS3603. 1999. *Timber Structures Standard*, New Zealand Standards, Wellington.

Palermo, A. 2004. "Use of Controlled Rocking in the Seismic Design of Bridges," Doctate Thesis, Technical Institute of Milan, Milan.

Palermo, A., Pampanin, S., Buchanan, A., and Newcombe, M. 2005. Seismic Design of Multi-Storey Buildings using Laminated Veneer Lumber (LVL). *2005 New Zealand Society of Earthquake Engineering Conference*, Wairaki, New Zealand, pp. 8.

Pampanin, S., and Marriott, D. 2010. NZCS PRESSS Design Handbook. New Zealand Concrete Society, Wellington, New Zealand.

Pampanin, S., Priestley, M. J. N., and Sritharan, S. 2001. Analytical Modelling of the Seismic Behavior of Precast Concrete Frames Designed with Ductile Connections. *Journal of Earthquake Engineering*, Vol. **5: 3**, pp. 329-367.

Pang, W., and Rosowsky, D. 2007. Direct Displacement Precedure for Performance-based seismic Design of Multistorey Woodframe Structures. *NEESWood Report NW-02*, Texas A&M University, College Station, <http://www.engr.colostate.edu/NEESWood/publications.html>.

Priestley, M. J. N. 1996. PRESSS program - current status and proposed plans for phase III. *PCI Journal*, Vol. **42: 2**, pp. 22-40.

Priestley, M. J. N., Calvi, G. M., and Kowalsky, M. J. 2007. *Displacement-Based Seismic Design of Structures*, IUSS PRESS, Pavia, Italy.

Priestley, M. J. N., Sritharan, S., Conley, J. R., and Pampanin, S. 1999. Preliminary Results and Conclusions from the PRESSS Five-story Precast Concrete Test-Building. *PCI Journal*, Vol. **44: 6**, pp. 42-67.

Rahman, A. M., and Restrepo, J. I. 2000. Earthquake Resistant Precast Concrete Buildings: Seismic Performance of Cantilever Walls Prestressed Using Unbonded Tendons. *Research Report 2000-5*, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand.

Smith, T., Fragiacomo, M., Pampanin, S., and Buchanan, A. H. 2009. Construction Time and Cost for Post-tensioned Timber Buildings. *Construction Materials*, Vol. **162: CM4**, pp. 141-149.

Smith, T., Ludwig, F., Pampanin, S., Fragiacomo, M., Buchanan, A., and Deam, B. 2007. Seismic response of Hybrid-LVL coupled walls under quasi-static and pseudo-dynamic testing. *NZSEE Conference, Palmerston North, New Zealand.*pp. 8.

Sullivan, T. J., Priestley, M. J. N., and Calvi, G. M. 2006. Seismic Design of Frame-Wall Structures. *Research Report No. 2006/02*, IUSS Press, Pavia, Italy.